SOIL INVESTIGATION FOR A PORTION OF THE PROPOSED SILVER CREEK AND THOMPSON CREEK IMPROVEMENTS

Zone E-1, Project No. 2

Santa Clara County, California

WOODWARD, CLYDE, SHERARD AND ASSOCIATES

Consulting Civil Engineers

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REPORT

to

SANTA CLARA COUNTY FLOOD CONTROL
AND WATER CONSERVATION DISTRICT
112 Hamline Street
San Jose, California

on

SOIL INVESTIGATION FOR A PORTION OF THE PROPOSED SILVER CREEK AND THOMPSON CREEK IMPROVEMENTS

Zone E-1, Project No. 2
Santa Clara County, California

by

WOODWARD-CLYDE-SHERARD & ASSOCIATES
Consulting Civil Engineers

Oakland, California

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Soil and Foundation Engineering

OAKLAND, CALIF.
DENVER, COLO.
OMAHA, NEB.
KANSAS CITY, MO.

February 25, 1958

Job No. 2239

Santa Clara County Flood Control
and Water Conservation District,
112 Hamline Street
San Jose, California

Attention: Mr. George H. Prime

Gentlemen:

In accordance with your request we have investigated the subsurface conditions at various locations along the proposed Silver Creek and Thompson Creek channel improvements, Zone E-1, Project No. 2 in Santa Clara County, California. The investigation was authorized by your letter of January 21, 1958.

The investigation indicates that soils of good supporting capacities are encountered at the channel crossings investigated. Although the soils are susceptible to some settlement under large heavily loaded areas, bridge foundations may be supported on spread footings. An alternate for a pile foundation is given for the case of the Lausett Avenue Bridge where negligible settlement may be desired. The soil within the depths of the proposed channel sections are generally impervious

The accompanying report includes the results of the subsurface exploration and the laboratory tests as well as the reasoning supporting the conclusions and recommendations contained therein..

The Staff Engineer assigned to this project by our firm was George Hervert.

Very truly yours,

WOODWARD-CLYDE-SHERARD & ASSOCIATES

By Stanley F. Gizienski
Stanley F. Gizienski, R.E. 10352

Reviewed By Richard J. Woodward
Richard J. Woodward, R.E. 8229

sr

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REPORT

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AND WATER CONSERVATION DISTRICT

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SILVER CREEK AND THOMPSON CREEK IMPROVEMENTS

Zone E-1, Project No. 2

Santa Clara County, California

SCOPE

This report describes an investigation of the subsurface conditions along a portion of the proposed alignment for the improvements to Silver Creek and Thompson Creek. The area investigated is located about 2 miles east of San Jose, California, from McKee and King Roads southeast to Lendrum Avenue. This study is intended to determine in part the most suitable type of foundation, required foundation depths and allowable soil pressures at a proposed bridge site. Since a considerable amount of channel excavation and levee construction will be done the problems relating to grading are also discussed.

In order to determine the subsurface conditions a series of test borings was made at the site. Soil samples were continuously examined during the drilling operations and visually identified in the field; undisturbed samples were recovered from the holes. The samples were subjected to visual inspection and laboratory tests in order to evaluate the physical and mechanical properties of the soils encountered. These properties were subsequently utilized in conjunction with observations on the site to determine the probable safe bearing values of the local soils and in reaching other conclusions and recommendations reported. For ease of reference a summary of findings follows; detailed discussions of the investigation and of the various factors influencing our recommendations are given in the body of the report.

SUMMARY

Lausett Avenue Bridge Site

1. At the bridge site the upper 30 ft of soil consist of firm to stiff sandy and silty clays; these are underlain by a layer of dense to very dense sand and gravel 15 ft thick. Stiff to very stiff sandy clay is found under this material to a depth of at least 58 ft. Free ground water at this site was encountered at depths of 9 and 11 ft below the ground or street surface 24 hours after drilling.

2. The proposed bridge may be supported on spread footings founded at a depth of at least 15 ft below the present ground surface; friction piles extending into the dense sands may also be used. If spread footings are used the allowable soil pressure should not exceed 3000 psf for dead plus live load and 4500 psf for all loads (including wind or seismic). The ultimate settlement of the abutment footings after construction is estimated to be of the order of 2.5 in.

3. If friction piles are used to support the bridge abutments they should extend 5 ft into the sand and gravel layer. Concrete piles having an average diameter of 12 in. driven to a depth of 34 ft below the existing road surface, or 25 ft below the bottom of the proposed channel, will have an allowable capacity of 40 tons.

4. Abutment walls retaining earth should be designed for an equivalent fluid pressure of 40 pcf plus an uniform surcharge of 100 psf. Backfill drains should be provided behind the walls as described in the body of the report.

Channel Crossing Sites and Channel Alignment

1. The soils at other locations generally consist of firm to stiff sandy or silty clays to depths of 20 to 35 ft. Thin layers of silty and clayey sand are encountered occasionally between the depths of 5 and 9 ft. Free ground water was encountered at depths of 12½ to 13 ft below the ground surface.

2. Material excavated from the channel is generally suitable for construction of levees. Silty sands encountered may be incorporated in the landward side of levees with the more impervious materials placed on the channel side.

3. The permeability of the native soils is estimated to range from $1\text{x}10^{-4}$ cm/sec for the silty sands to about $1\text{x}10^{-7}$ cm/sec for the stiff silty clays.

4. The silty and clay soils are relatively low in susceptibility to erosion. The silty sands encountered at depths of 5 to 6½ ft in the area of the Lausett Avenue site and at 5½ to 9 ft at Hole 4 are susceptible to some erosion by swiftly flowing water.

FIELD INVESTIGATION

Five test holes were drilled at the locations shown on Figs. 1 through 3. A 6-in. diameter power auger was used for advancing the holes. Undisturbed samples were taken with a 2-in. diameter modified California sampler. Samples were driven with a 140-lb weight which was given a free fall of 30 in.; resistance to penetration of the sampler was observed. Additional penetration tests were made in Hole 2 by driving a 2¼-in. diameter steel shoe with a 280-lb hammer falling 30 in. The drilling was done on January 27, 29 and February 2, 1958.

The depths at which undisturbed samples were taken in the test holes are shown on the logs of borings, Figs. 4 through 7. Samples are designated first according to the hole from which they were recovered and then according to their vertical location in the hole; the top sample in a hole is numbered 1, the next one down is 2, and so on. As an example, the third sample taken from Hole 2 would be numbered 2-3. Each sample is thus identified throughout the laboratory tests.

LABORATORY TESTS

The water content, dry density and unconfined compressive strength were determined for each sample if possible. Grain size analyses were made on typical samples of surface soil taken from the banks and borings in order to determine the amounts of sand, silt, clay and colloidal sizes present; liquid, plastic and shrinkage limits were also determined for these samples in order to classify them with respect to plasticity characteristics. Two laboratory compaction tests were made on typical sack samples in order to determine the compaction characteristics.

The results of these tests are given in Table 1. The resistance to penetration by the sampler or the steel shoe, water content, dry density and unconfined strength are also shown at the corresponding sample location on the logs of borings, Figs. 4 through 7. Classification of the samples according to plasticity characteristics is shown on Fig. 8.

A consolidation test was performed on a typical sample of the more compressible soils encountered at the Lausett Avenue bridge site. Results of this test are shown in Fig. 9 in the form of pressure-void ratio and time-percent consolidation curve.

In order to determine the shear strength of material expected to be used as a backfill behind abutment walls, three similar samples were tested by triaxial compression under varying degrees of confinement. The results of these tests are shown in the form of Mohr circles in Fig. 10.

PROJECT AND SITE DESCRIPTION

The proposed improvements consist of straightening and widening of the existing channel. Portions of the channel will be lined with concrete while other portions are to be unlined. Excavated material from the channel will be used to construct small levees on the creek banks. At Lausett Avenue a four-lane bridge designed for H-15-512-44 Highway loading, is to be constructed.

The general area within the boundaries of the improvements is level. The inclinations of the existing banks vary from nearly vertical to 2 to 1 (2 horizontal to 1 vertical). In some locations fences or sheds are very close to the existing right-of-way. Maximum flow through the channel is expected to approach 4500 cfs.

SOIL CONDITIONS

Lausett Avenue Bridge Site - The borings show that the soils in the upper 30 ft consist generally of firm to stiff sandy to silty clays; a loose to medium dense silty sand layer is encountered at a depth of 5 to 6½ ft. The soils below a depth of about 14 ft are stiff. From 30 to 45 ft a dense to very dense layer of sand and gravel is encountered; this is underlain, in turn, by stiff to very stiff sandy clay to a depth of at least 60 ft. Free ground water was encountered at a depth of 9 to 11 ft below the abutment ground surface at the time of drilling. A typical profile at the bridge site is shown in Fig. 11.

The surface soils are medium in plasticity and are high in swelling potential.

The clays above a depth of 14 ft are more compressible than those encountered between the depths of 14 and 30 ft; below 30 ft the sands and gravels are relatively incompressible.

Lendrum Avenue - The soils in this area are generally firm to stiff sandy or silty clays to a depth of at least 20 ft. Water was encountered 12 ft below the ground surface at the time of drilling.

End of St. James Street - About 3½ ft of fill has been placed in this area. The soils consist generally of firm to stiff sandy or silty clays. A layer of loose to medium dense silty sand is encountered between 5½ and 9 ft. Free water was encountered 13 ft below the ground surface at the time of drilling.

Intersection of McKee and King Roads - The upper 3 ft of soil is a fill consisting of stiff sandy clay and loose to medium dense gravel and sandy clay. The underlying soils to a depth of 35 ft are firm to stiff silty or sandy clays containing occasional lenses of sand. Below 35 ft dense silty sand containing gravel is encountered. The water table was encountered 12½ ft below the ground surface at the time of drilling.

DISCUSSION

Two requirements must be fulfilled by any foundation material. First it must be safe against shear failure, which would result in lateral movement of soil from under the load. Second, the vertical settlement must not exceed the amount permissible for the particular type of structure. In addition, a bridge foundation must be safe against undermining by scour.

Results of tests on and experiences with loads resting on clay soils have indicated that, as far as lateral shear failure is concerned, the ultimate bearing capacity can be determined from the results of unconfined compression tests. Both theory and experience indicate that the average value of the shearing stress on the surface of impending failure for a clay in the field is approximately equal to one-half the unconfined compressive strength. Theoretically the ultimate bearing capacity of a clay soil is equal to about 5 or 6 times the shear strength. Thus, with the inclusion of an adequate factor of safety, a satisfactory rule for a clay soil is not to allow the vertical pressure on a stratum to exceed the unconfined compressive strength (except for short time loads).

Settlement on clay soils are usually estimated from the results of consolidation tests. Sandy or gravelly soils have adequate shear strengths even though they show low unconfined compressive strengths. Settlement on such soils is not excessive if the bearing value is governed by the compactness of the material. Compactness is best measured by the resistance to penetration of the sampler or other similar device.

Lausett Avenue Bridge - Below a depth of 14 ft stiff clays having good compressive strengths are encountered at the bridge site. These soils are capable of supporting the bridge foundation without danger of shear failure. Since a wide heavily loaded footing is required settlement characteristics rather than stability will govern.

Based on the results of the consolidation test it is estimated that if spread footings are used ultimate settlements of the order of 2.5 in. can be expected after completion of construction. Settlement computations assume an 8 ft wide long footing founded at a depth of 15 ft and subjected to a dead-load soil pressure of 3000 psf. At least 50 percent of this settlement is expected to occur during the first 10 years after construction; the settlement should be uniform on both sides of the channel.

Alternate I - Spread Footings - It is our opinion that the proposed bridge can be safely supported on spread footings founded at a depth of 15 ft below the existing road surface providing the bridge superstructure is capable of withstanding about 2½ in. of settlement. The soil pressure under spread footings should not exceed 3000 psf for dead and live loads and 4500 psf for all loads (including wind or seismic). The recommended depth will place the footings about 7 ft below the existing channel; there should be little danger of scour since the channel beneath the bridge will be paved with concrete.

Alternate II - Piles - A compact granular layer 15 ft thick is encountered below a depth of about 30 ft, therefore, piles extending into this layer can be used to support the structure with negligible settlement resulting. Theoretically the ultimate bearing capacity of a pile driven into a truly cohesive clay soil is equal to the effective surface area of the pile times one-half the unconfined compressive strength; this is a conservative approach since the soils also develop increasing frictional resistance with depth. The dense granular layer below 30 ft will provide additional end bearing support to the pile.

Based on the laboratory test results and field penetration tests it is estimated that a pile with an average diameter of 12 in. and which penetrates at least 5 ft into the sand and gravel layer will develop an allowable capacity of 40 tons. The piles should extend 25 ft below the present channel or 34 ft below the surface of the abutments. Such piles would have a factor of safety of at least 2. Pipe, precast or cast-in-place concrete piles may be used. The type of pile selected should be based on economic considerations and ease of driving. Cast-in-place piles are less difficult to drive in granular layers and are easily adjustable to changes of design length.

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Abutments - Abutment walls retaining earth should be provided with backfill drains to avoid accumulation of water behind the walls.. For your convenience a "Guide Specifications for Subsurface Drains" is enclosed. The results of triaxial tests on abutment materials show that the angle of internal friction is 28 degrees. It is recommended that the walls be designed for an equivalent fluid pressure of 40 pcf plus an uniform surcharge due to traffic of 100 psf.

Channel Crossing Sites and Channel Alignment - Soils along the proposed channel improvement consist essentially of clays; therefore materials excavated from the channel may be used for levee construction. In certain reaches such as in the areas of the Lausett Avenue bridge and at our Hole 4, layers of silty sand are encountered. These materials should be used in the landward side of levees.

The silty sand layers are also susceptible to erosion by high stream velocities. Channel design should consider avoiding high velocities or else lining the channel in such areas.

Compaction tests were made to determine the maximum dry unit weight of the soil by the California Impact Compaction Test Procedure. A representative sample of silty clay had a maximum dry density of 112 pcf and optimum moisture of 16 percent; the sandy clay had a maximum dry density of 117 pcf and an optimum moisture content of 14 percent. The results are given in Table 1. Proper control and compaction of these soils in the field will provide stable material for levees.

The natural soils are quite stratified and somewhat variable; hence laboratory determinations of the coefficient of permeability were not made. Based on our experience with tests on similar materials it is estimated that the coefficient of permeability of the silty and sandy clays will be of the order of $1x10^{-7}$ cm/sec; these soils may be classed as being very low in permeability. The silty sands are estimated to have a coefficient of permeability of about $1x10^{-4}$ cm/sec; they may be classed as low to medium in permeability.

The test holes, with the exception of those at the bridge site, were drilled at locations requested by others and are spaced at considerable distances apart. In cases where holes are spaced far apart it is not possible to assume a uniform variation of materials between test holes. If any unusual conditions are encountered during construction please feel free to call on us.

This report is issued with the understanding that it is the responsibility of the owner to ensure that the applicable provisions of the recommendations contained herein are called to the attention of the Architect and Engineers and incorporated into the plans and that the necessary steps be taken to see that the Contractor and subcontractors carry out such provisions in the field.

TABLE 1 - SUMMARY OF TEST RESULTS

SAMPLE NUMBER	SAMPLE DEPTH, FT.	WATER CONTENT, % DRY WT.	DRY DENSITY, PCF	UNCONFINED COMPRESSIVE STRENGTH, PSF	STANDARD PENETRN., BLOWS/FT.	CLASSIFICATION
1-1	4½	16	100	6590	13	Silty clay
1-2	9½	26	105	-	9	Silty clay
1-3	20	28	97	6000	28	Silty clay
2-1	3½	14	98	3710	9	Silty clay
2-2	8½	26	99	-	11	Silty clay
2-3	13½	32	91	3710	8	Silty clay
2-4	24	26	99	7120	17	Silty clay
3-1	3	15	107	8300	14	Clay
3-2	5½	13	109	5120	10	Clayey sand
3-3	9	28	98	3270	9	Clayey silt
3-4	19	28	97	6340	15	Silty clay
3-5	29½	19	111	1610	12	Gravelly clay
4-1	4½	15	101	5610	15	Silty clay
4-2	9½	3	-	-	9	Silty sand
4-3	14½	21	104	-	9	Silty sand (Fall In)
4-4	20	22	107	1560	12	Sandy clay
5-1	3	15	97	-	17	Sandy clay
5-2	8½	20	108	6980	9	Silty clay with sand
5-3	14½	24	97	6540	10	Silty clay
5-4	24½	27	99	1460	7	Silty clay
5-5	33	25	102	2540	12	Silty clay
5-6	40	17	109	490	52	Silty sand with gravel

TABLE 1 - SUMMARY OF TEST RESULTS (CONTINUED)

	SAMPLE NUMBER	
ATTERBERG LIMITS	Sack Sample 2-1	Sack Sample 5-2
Liquid Limit, % Dry Weight	33	45
Plasticity Index	15	21
Shrinkage Limit	9.6	17.0
GRAIN SIZES, % DRY WEIGHT		
Sand-Silt-Clay (Colloidal)	32-31-37(23)	3-39-58(30)
OPTIMUM MOISTURE CONTENT, %*	14	16
MAXIMUM DRY DENSITY, PCF*	117	112

*California Impact Compaction Test

GUIDE SPECIFICATIONS FOR SUBSURFACE DRAINS

(a) DESCRIPTION

Underdrains consisting of standard drain tile, perforated Orangeburg drain pipe, or perforated galvanized corrugated metal pipe shall be installed as shown on the plans and in accordance with these specifications, unless otherwise specified by the Engineer.

(b) MANUFACTURE

Underdrains shall be manufactured in accordance with the following requirements:

(1) Standard drain tile shall conform to the requirements of ASTM Designation C4.

(2) Perforated corrugated metal pipe shall conform to the AASHO Designation M136.

(3) Perforated Orangeburg pipe shall conform to the Federal Specifications SS-P-356.

(c) FILTER MATERIAL

Filter material for use in backfilling trenches around and over underdrains and for pervious blankets shall consist of clean, coarse sand and gravel or crushed stone conforming to one of the following grading requirements:

Sieve Size	Percentage Passing Sieves		
	Type A	Type B	Type C
2½"			100
1½"		100	80-100
¾"	100	90-100	60-95
No. 4	80-100	55-85	35-65
No. 8	60-90	35-65	25-50
No. 30	20-50	15-35	5-25
No. 50	10-25	10-25	
No. 200	0-4	0-3	0-3

Unless otherwise specified, Type B filter material shall be used.

(d) LAYING

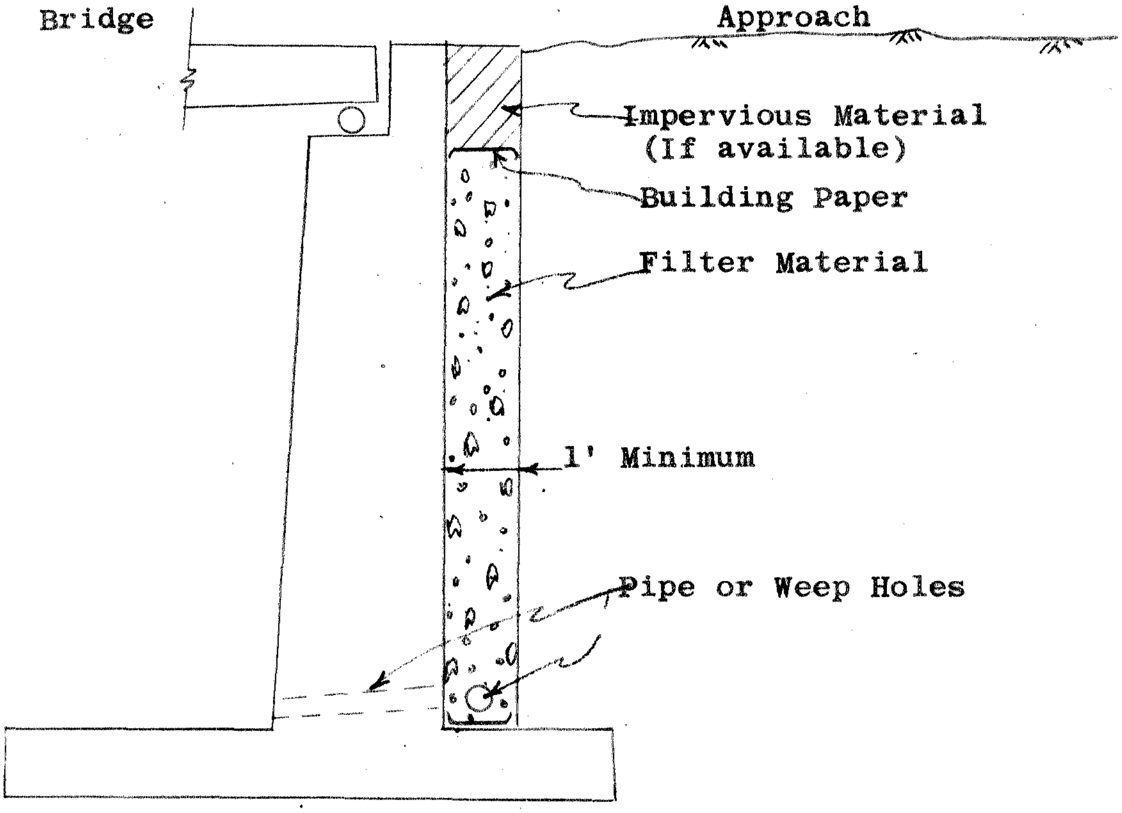
Trenches for underdrains shall be excavated to a minimum width equal to the outside diameter of the pipe plus one foot and to the depth shown on the plans or as directed by the Engineer. The bottom of the trench shall then be covered full width by one layer of plastic building paper. The drain pipe shall then be laid directly on the building paper.

GUIDE SPECIFICATIONS FOR SUBSURFACE DRAINS (CONTINUED)

Unless otherwise specified perforated pipe shall be laid with the perforations at the bottom and sections shall be joined with couplers. Drain tile shall be laid with a clear opening of ¼" between sections.

After the pipe has been placed, the trench shall be backfilled with filter material to the elevation shown on the plans, or as directed by the Engineer. The filter rock shall then be covered with a layer of building paper or plastic. The remainder of the trench shall then be backfilled with an impervious soil which shall be compacted to the requirements for engineered fill.

TYPICAL SECTION



Recommended Drain Behind Bridge Abutment

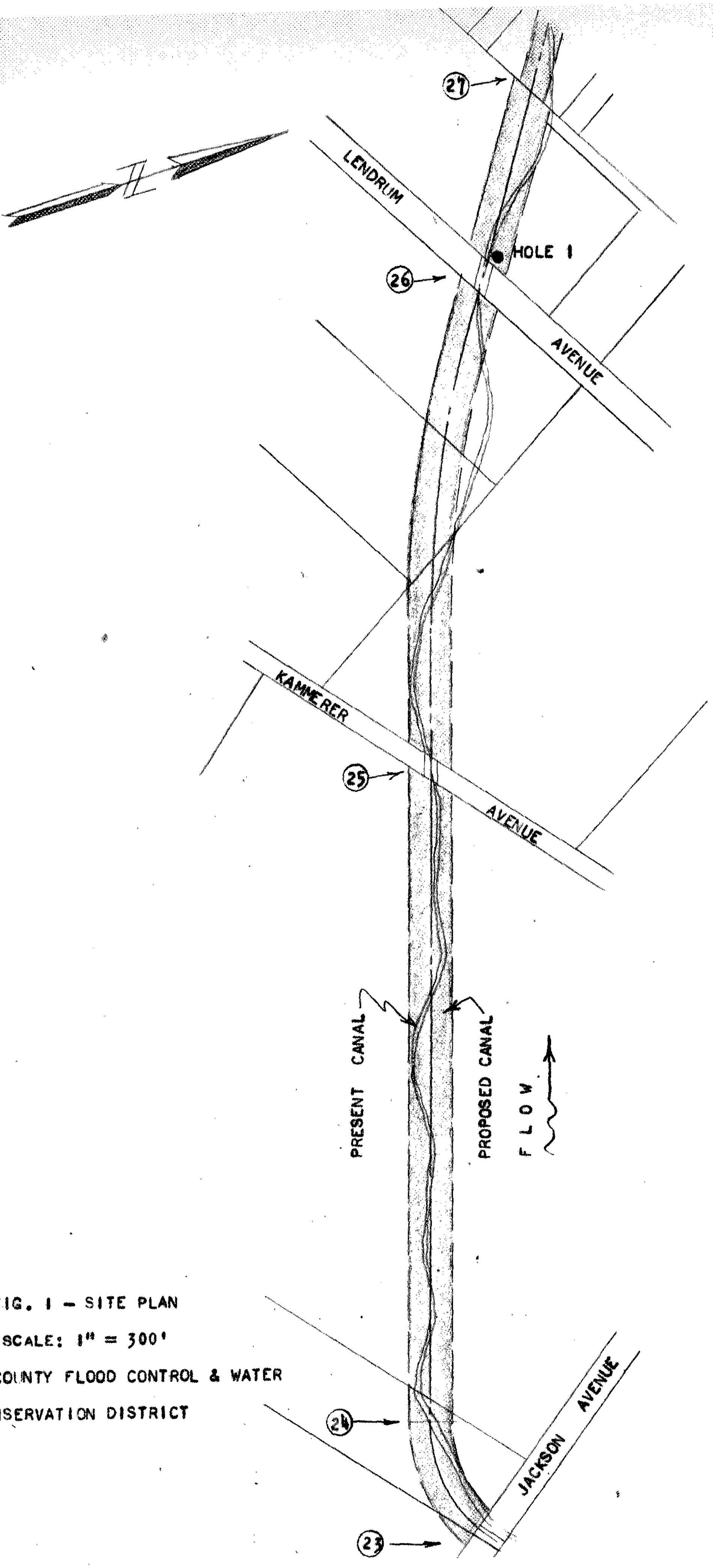


FIG. 1 – SITE PLAN

SCALE: 1" = 300'

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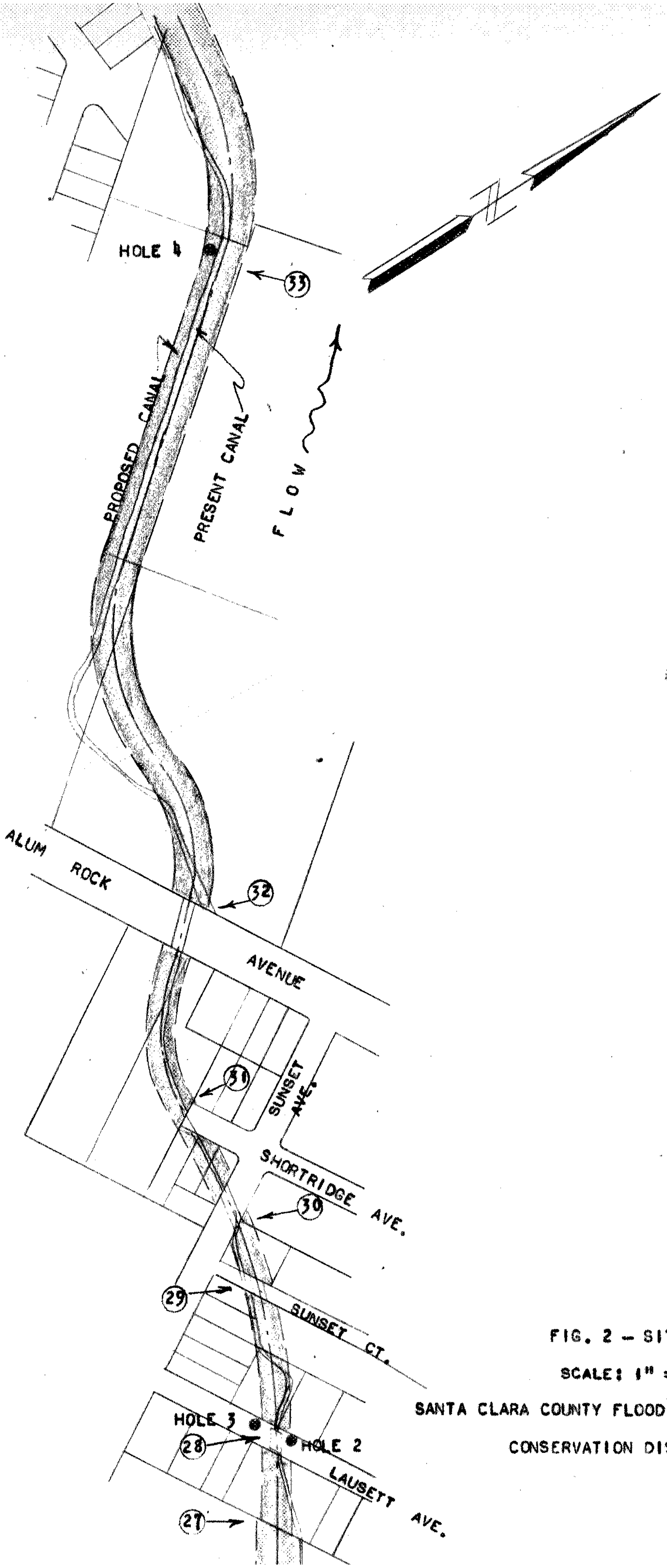

FIG. 2 – SITE PLAN

SCALE: 1" = 300'

SANTA CLARA COUNTY FLOOD CONTROL & WATER CONSERVATION DISTRICT

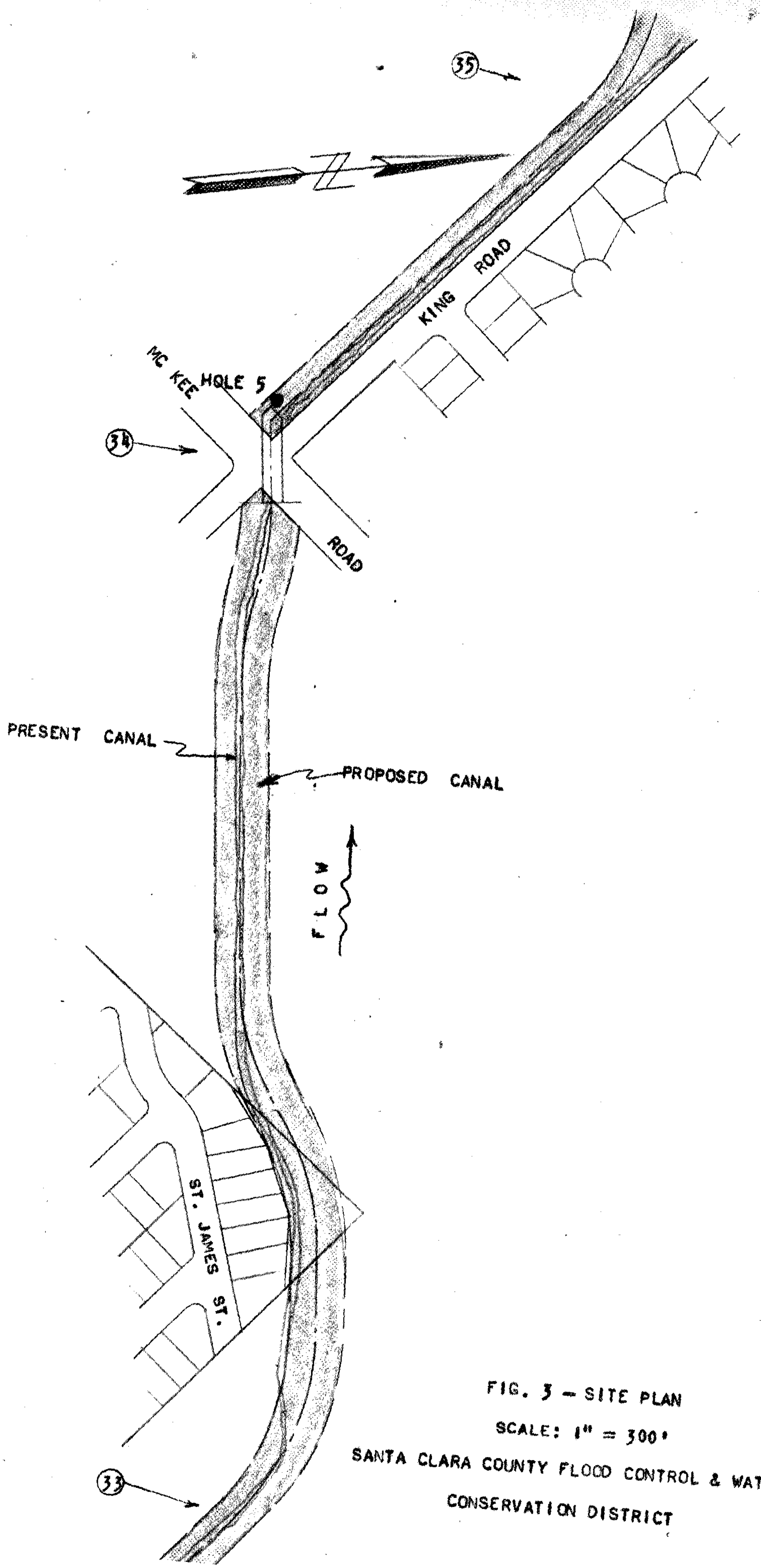

FIG. 3 – SITE PLAN

SCALE: 1" = 300'

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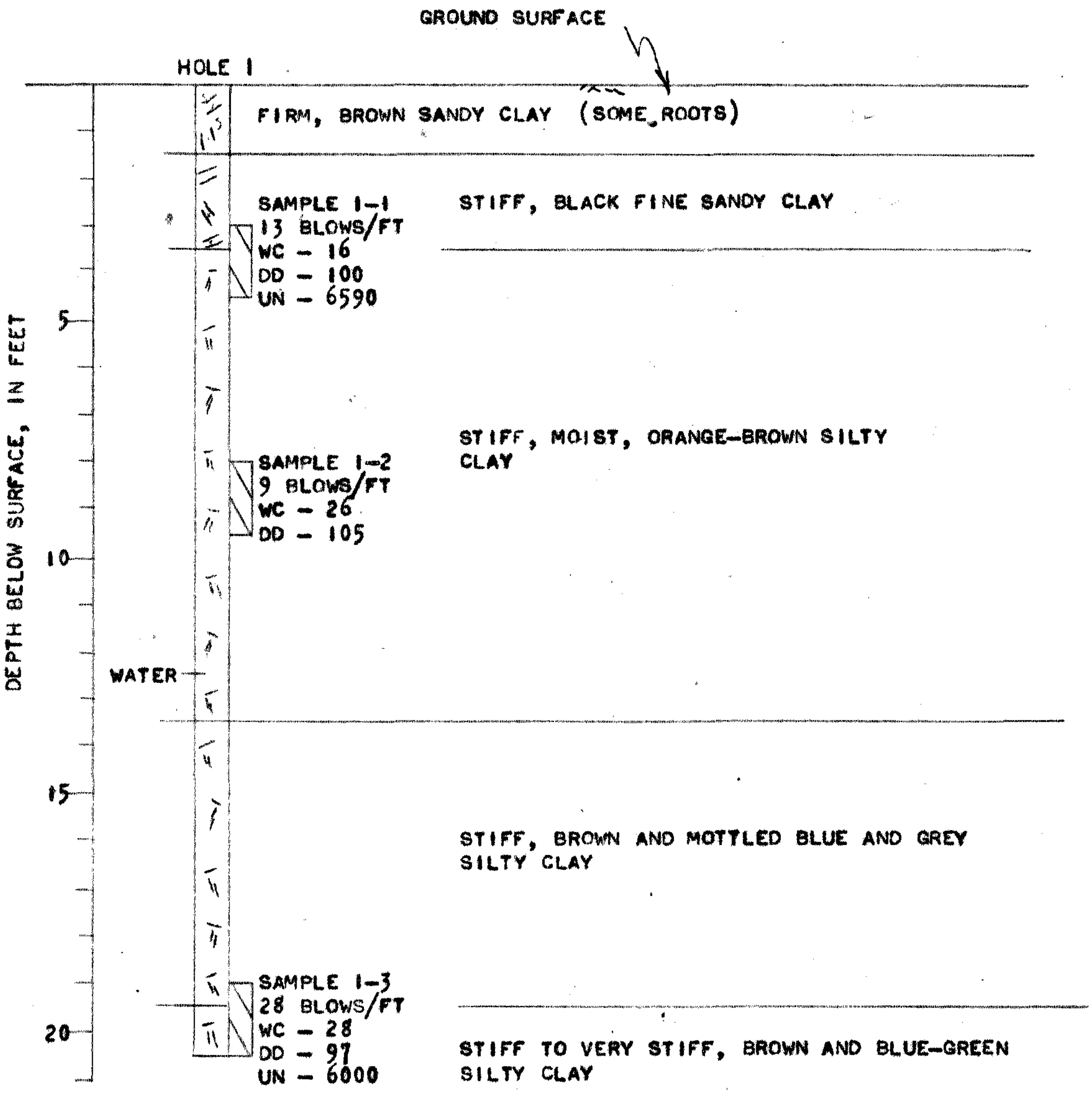
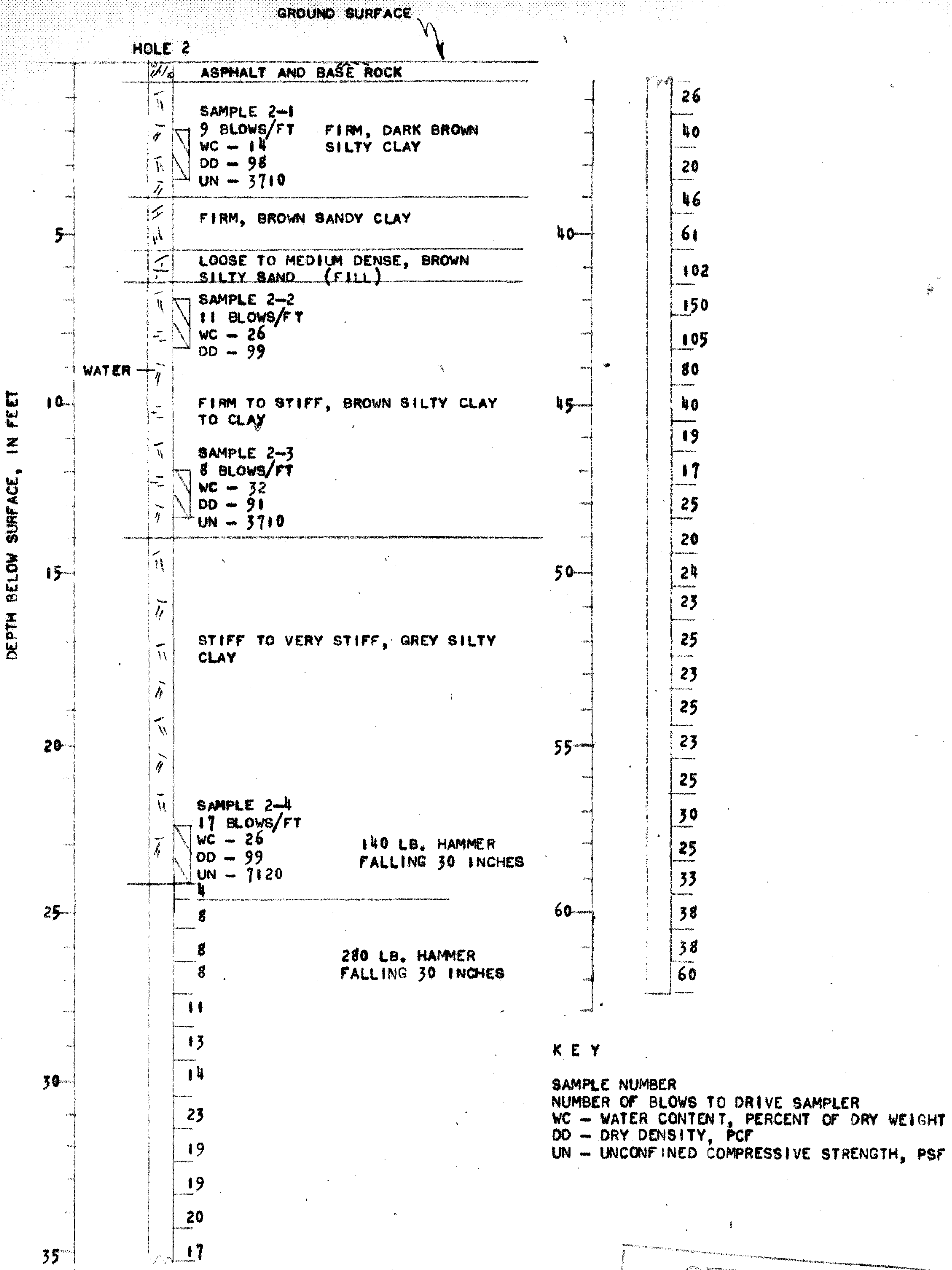

GROUND SURFACE
HOLE 1
FIRM, BROWN SANDY CLAY (SOME ROOTS)
SAMPLE 1-1
13 BLOWS/FT
WC - 16
DD - 100
UN - 6590
STIFF, BLACK FINE SANDY CLAY
STIFF, MOIST, ORANGE-BROWN SILTY CLAY
SAMPLE 1-2
9 BLOWS/FT
WC - 26
DD - 105
WATER
STIFF, BROWN AND MOTTLED BLUE AND GREY SILTY CLAY
SAMPLE 1-3
28 BLOWS/FT
WC - 28
DD - 97
UN - 6000
STIFF TO VERY STIFF, BROWN AND BLUE-GREEN SILTY CLAY
DEPTH BELOW SURFACE, IN FEET
5
10
15
20

KEY

SAMPLE NUMBER
NUMBER OF BLOWS TO DRIVE SAMPLER
WC - WATER CONTENT, PERCENT OF DRY WEIGHT
DD - DRY DENSITY, PCF
UN - UNCONFINED COMPRESSIVE STRENGTH, PSF

FIG. 4 - LOG OF BORINGS



K E Y

SAMPLE NUMBER
NUMBER OF BLOWS TO DRIVE SAMPLER
WC - WATER CONTENT, PERCENT OF DRY WEIGHT
DD - DRY DENSITY, PCF
UN - UNCONFINED COMPRESSIVE STRENGTH, PSF

FIG. 5 - LOG OF BORINGS

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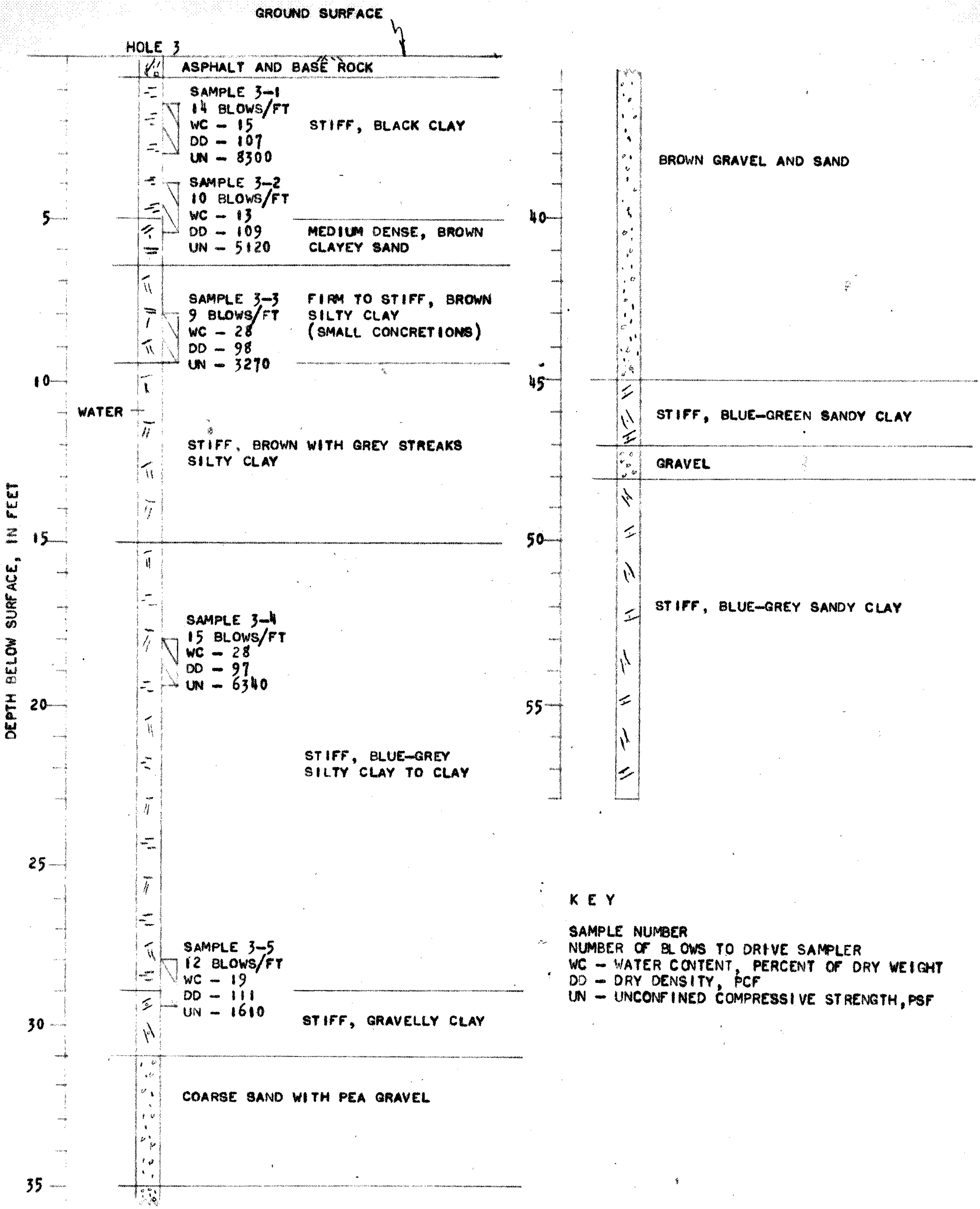


FIG. 6 — LOG OF BORINGS

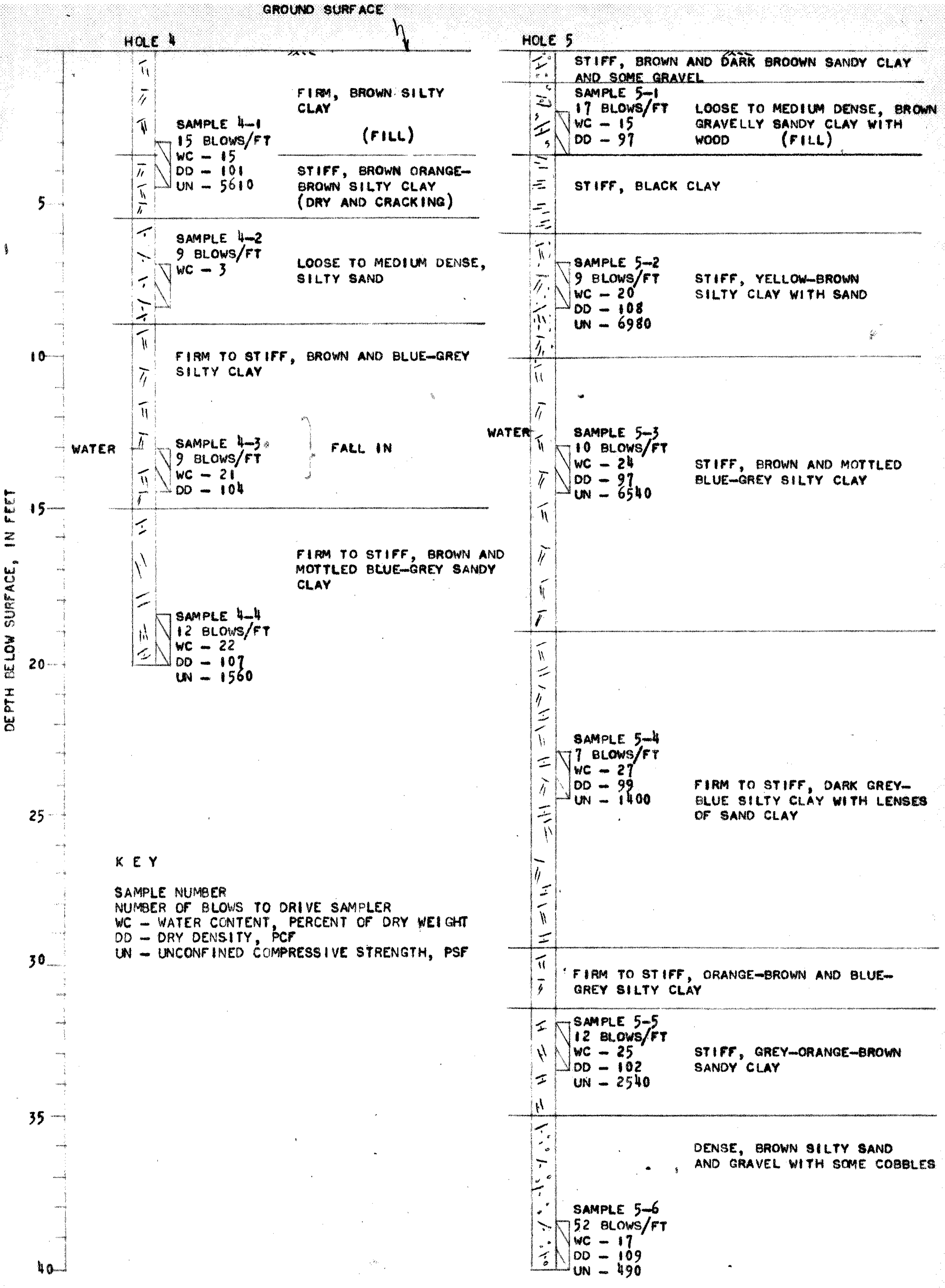

GROUND SURFACE
HOLE 4
HOLE 5
FIRM, BROWN SILTY CLAY
(FILL)
SAMPLE 4-1
15 BLOWS/FT
WC - 15
DD - 101
UN - 5610
STIFF, BROWN ORANGE-BROWN SILTY CLAY (DRY AND CRACKING)
SAMPLE 4-2
9 BLOWS/FT
WC - 3
LOOSE TO MEDIUM DENSE, SILTY SAND
FIRM TO STIFF, BROWN AND BLUE-GREY SILTY CLAY
WATER
SAMPLE 4-3
9 BLOWS/FT
WC - 21
DD - 104
FALL IN
FIRM TO STIFF, BROWN AND MOTTLED BLUE-GREY SANDY CLAY
SAMPLE 4-4
12 BLOWS/FT
WC - 22
DD - 107
UN - 1560
STIFF, BROWN AND DARK BROOWN SANDY CLAY AND SOME GRAVEL
SAMPLE 5-1
17 BLOWS/FT
WC - 15
DD - 97
LOOSE TO MEDIUM DENSE, BROWN GRAVELLY SANDY CLAY WITH WOOD (FILL)
STIFF, BLACK CLAY
SAMPLE 5-2
9 BLOWS/FT
WC - 20
DD - 108
UN - 6980
STIFF, YELLOW-BROWN SILTY CLAY WITH SAND
WATER
SAMPLE 5-3
10 BLOWS/FT
WC - 24
DD - 97
UN - 6540
STIFF, BROWN AND MOTTLED BLUE-GREY SILTY CLAY
SAMPLE 5-4
7 BLOWS/FT
WC - 27
DD - 99
UN - 1400
FIRM TO STIFF, DARK GREY-BLUE SILTY CLAY WITH LENSES OF SAND CLAY
FIRM TO STIFF, ORANGE-BROWN AND BLUE-GREY SILTY CLAY
SAMPLE 5-5
12 BLOWS/FT
WC - 25
DD - 102
UN - 2540
STIFF, GREY-ORANGE-BROWN SANDY CLAY
DENSE, BROWN SILTY SAND AND GRAVEL WITH SOME COBBLES
SAMPLE 5-6
52 BLOWS/FT
WC - 17
DD - 109
UN - 490
DEPTH BELOW SURFACE, IN FEET
5
10
15
20
25
30
35
40
KEY
SAMPLE NUMBER
NUMBER OF BLOWS TO DRIVE SAMPLER
WC - WATER CONTENT, PERCENT OF DRY WEIGHT
DD - DRY DENSITY, PCF
UN - UNCONFINED COMPRESSIVE STRENGTH, PSF

FIG. 7 — LOGS OF BORINGS

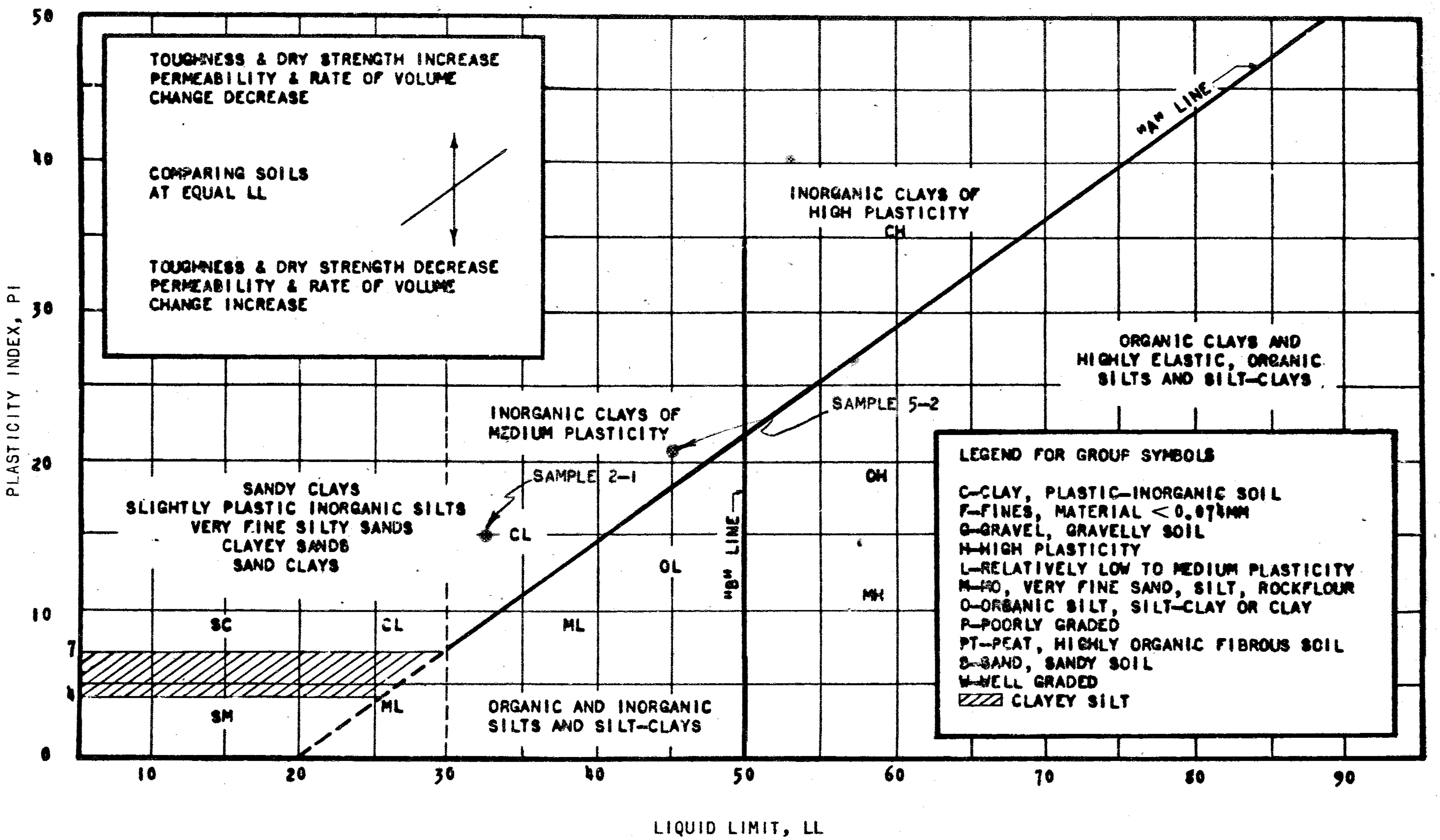
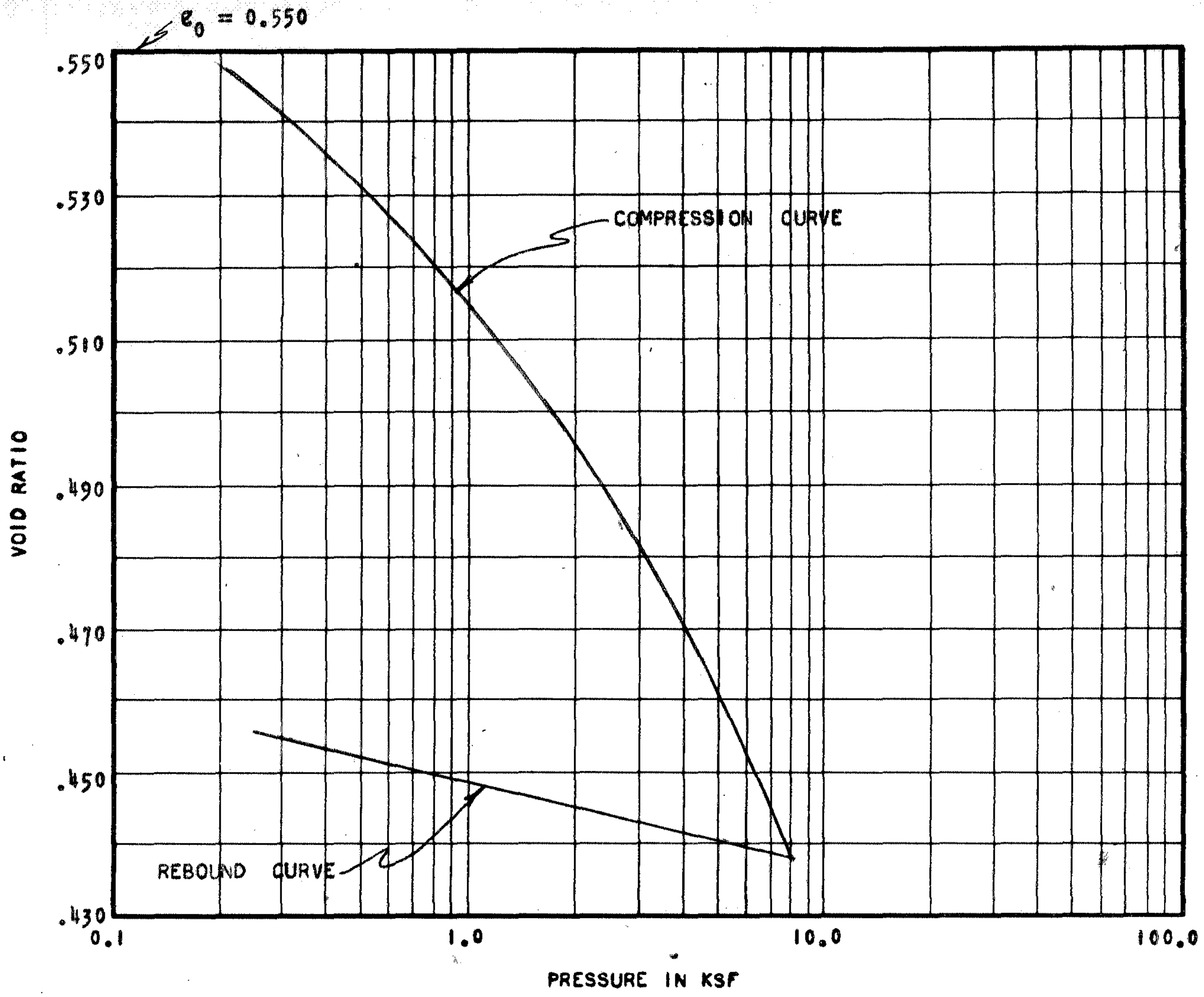


FIG. 8 -PLASTICITY CHART



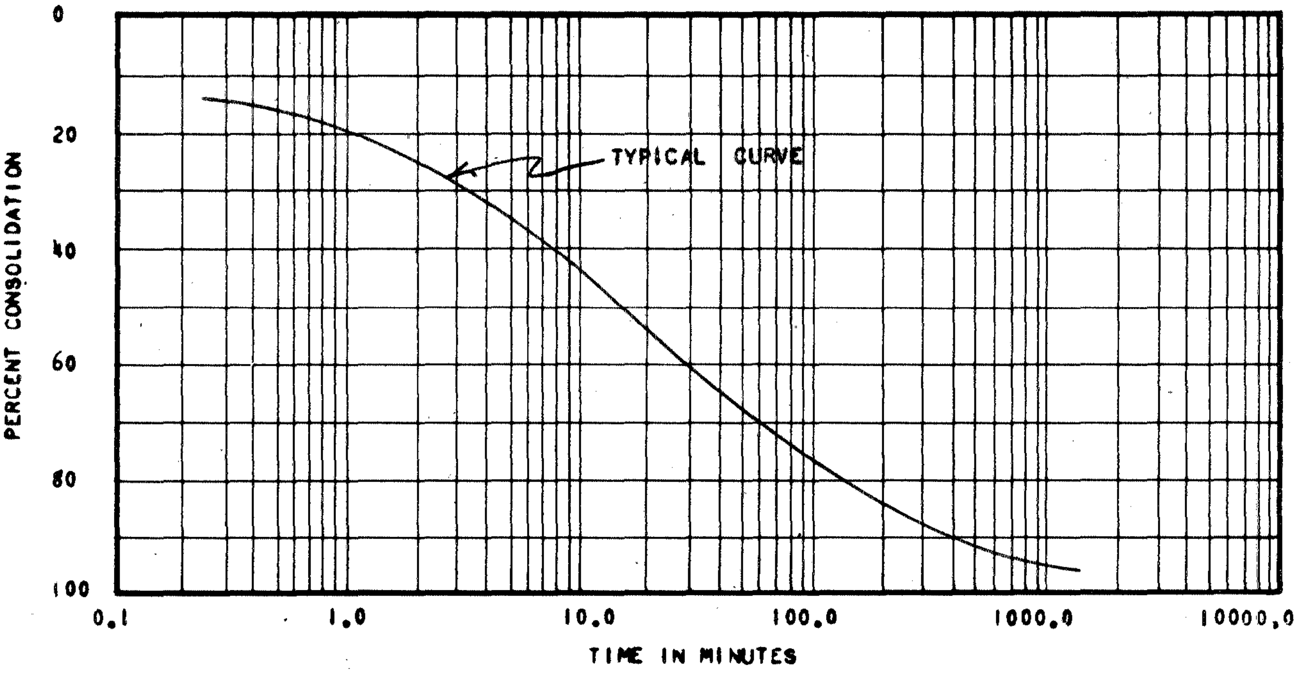


FIG. 9 — CONSOLIDATION TEST RESULTS — SAMPLE 3–5

CONSOLIDATED DRAINED

INITIAL CONDITION

SPECIMEN NO.	SAMPLE NO.	MOISTURE CONTENT, %	DRY DENSITY PCF	CONFINING LATERAL PRESSURE, PSF	DEVIATOR STRESS, PSF
1	2-2-3	28	95	500	860
2	2-2-4	26	99	1000	2000
3	3-3-3	25	96	2000	3600

ANGLE OF INTERNAL FRICTION $\phi = 27.9^0$

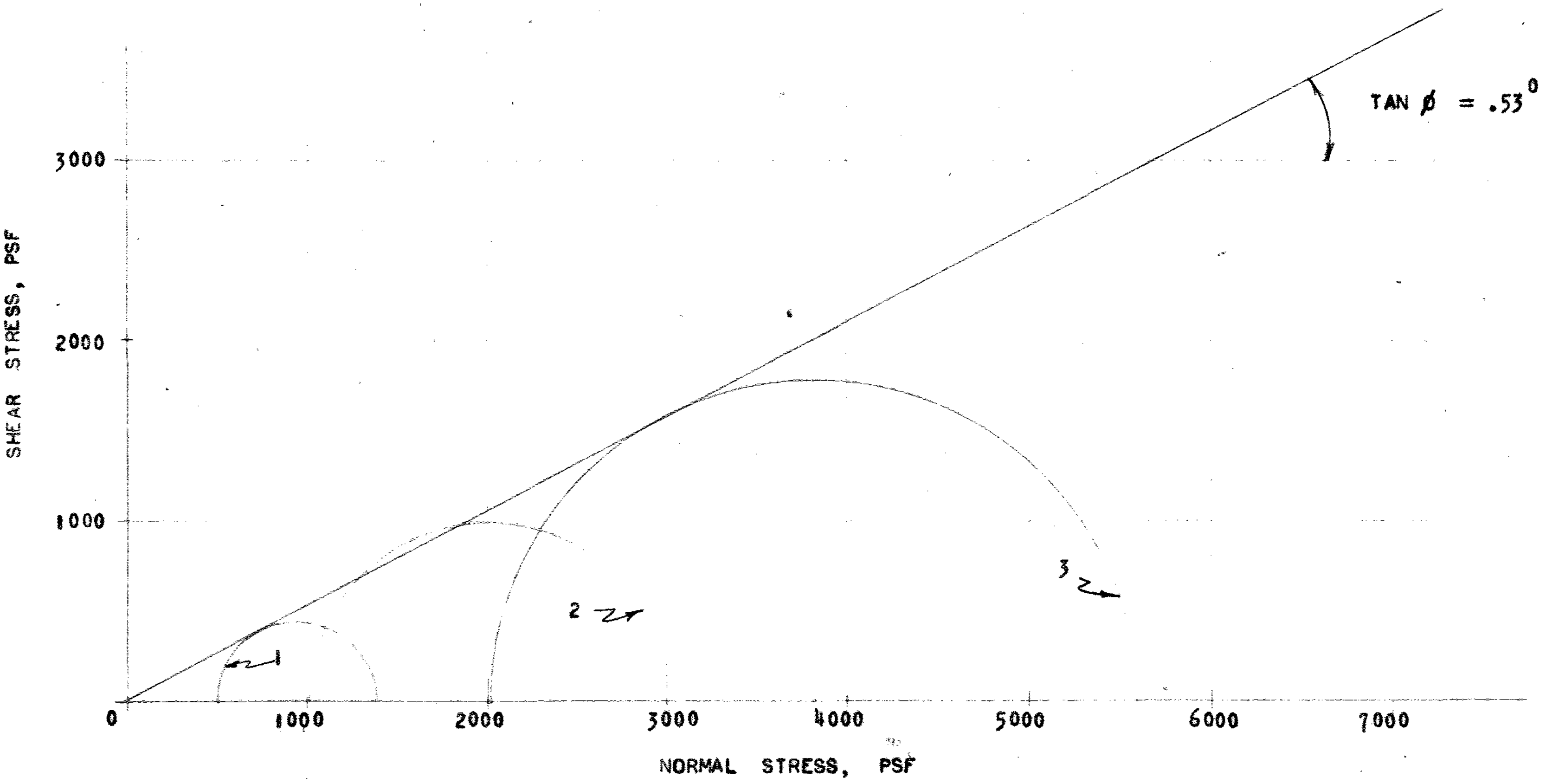


FIG. 10 — TRIAXIAL SHEAR TEST — CONSOLIDATED DRAINED

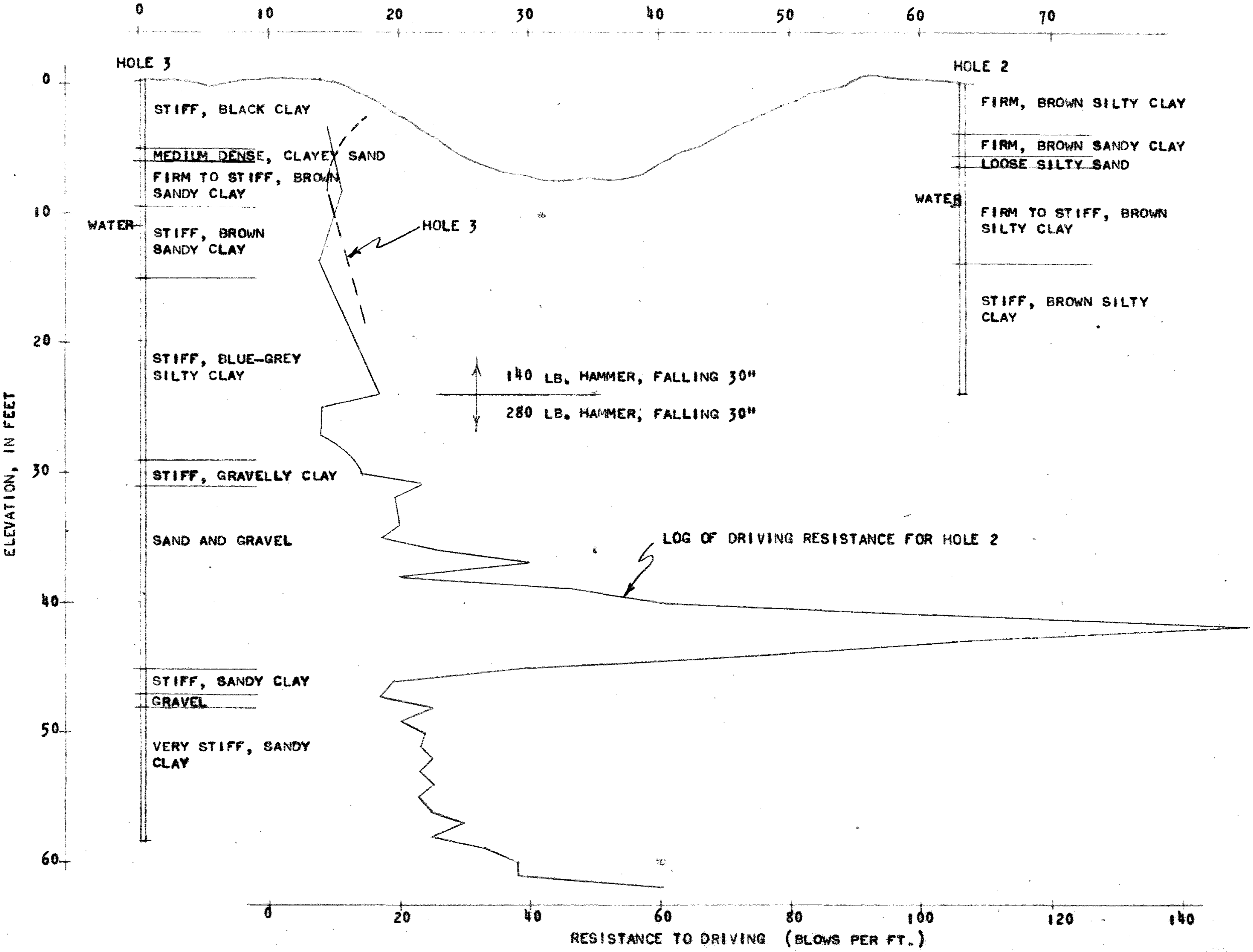


FIG. 11 – PROFILE AT BRIDGE SITE (LAUSETT AVENUE)